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SOIL MECHANICS AND BITUMINOUS MATERIALS RESEARCH LABORATORY

ASPHALT MIXTURE BEHAVIOR IN REPEATED FLEXURE

by
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Institute of Engineering Research
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The Materials and Research Department
Division of Highways
State of California

Prepared in Cooperation with the United States
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DEPARTMENT OF CIVIL ENGINEERING
INSTITUTE OF TRANSPORTATION AND TRAFFIC ENGINEERING

University of California · Berkeley

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Soil Mechanics and Bituminous Materials
Research Laboratory

ASPHALT MIXTURE BEHAVIOR IN REPEATED FLEXURE

A Report on an Investigation
by C. L. Monismith
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to

The Materials and Research Department
Division of Highways
State of California
Under
State of California Standard Agreement MR-127

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INTRODUCTION

To better define the fatigue behavior of asphalt mixtures resulting from repeated flexure, a four-year program is currently underway at the Soil Mechanics and Bituminous Materials Laboratory of the University of California supported by a research grant from the Materials and Research Department of the California Division of Highways.

In general, the initial objectives of this project have been to define the influence of aggregate type, and asphalt source and hardness, on the fatigue characteristics of asphalt concrete conforming to current State of California specification requirements.[1] From the results of the investigation during the first year,[2] the following trends were noted:

1. Asphalt type would appear to affect the fatigue results in constant-strain amplitude tests.
2. For asphalts from the same source, the initial hardness would appear not to affect the strain vs. cycles-to-failure relationship in constant-strain amplitude tests at a particular temperature.
3. The density of specimens (and indirectly the air void content) apparently affects test results, with longer life associated with higher densities (lower air void contents). This relationship may not be a direct proportion, however, since at higher void contents the possibilities of stress concentrations exist.
4. To interpret the results of constant-strain fatigue tests, the stiffness of the material under the particular conditions of loading as well as the strain vs. cycles-to-failure relationship would appear to be necessary.
5. Aggregate type and grading may affect the strain vs. cycles-to-failure relationship.

This report is the second annual report and covers research performed on the project during the period November 1963 through October 1964. In general, the report is concerned with the development of additional data to meet the initial objectives of the program, i.e., the influence of material characteristics on fatigue behavior of paving mixtures. In addition, some data are presented illustrating the effect of mixture composition (primarily asphalt content) on fatigue behavior, and some thoughts are presented relative to the relationship of laboratory measured properties to the performance of actual pavements in the field.

MATERIALS AND SPECIMENS

Constant-strain amplitude fatigue tests were conducted with beams fabricated from two aggregates, Watsonville granite and Cache Creek uncrushed gravel. The

Cache Creek gravel samples were supplied by the Materials and Research Department of the Division of Highways. Grading curves for both materials which conform to the 1/2 maximum medium gradation of the 1964 State of California Standard Specifications are illustrated in Fig. 1.

In the original stocks of Cache Creek gravel (uncrushed) supplied by the Materials Department, there was a deficiency of material passing the No. 50 sieve. Subsequent samples supplied were composed of crushed material rather than the uncrushed gravel. Thus, results for approximately one-half of the beam specimens reported herein are based on tests on specimens using crushed material as the material passing the No. 50 sieve. From stiffness measurements, however, there appears to be no effects as compared to the use of uncrushed material passing the No. 50 sieve so that all results can be interpreted within the same framework.

Weight-volume relationships for the mixtures were determined using an aggregate specific gravity of 2.92 for those specimens containing the Watsonville granite, and the apparent specific gravity values shown in Table 1 for specimens containing Cache Creek gravel.

In the majority of the tests reported herein, asphalts supplied to the State by the Standard Oil Company of California (referred to in the previous report as the S-Series) were utilized. It was necessary to obtain additional quantities of each of these asphalts because of the large number of specimens tested. Fortunately, the new materials were blended by the staff at California Research Corporation to have essentially the same properties as the original asphalts (samples of the original materials were furnished) and the results can thus be compared with earlier test series. Test results from the original samples are compared with results for the new materials in Table 2.

Mixture design tests for specimens containing the Watsonville granite and the S-Series asphalts are illustrated in Fig. 2. It will be noted that the stability values drop more sharply for the specimens containing the 40-50 penetration asphalt cement than for the specimens containing the softer asphalts. This may be due in part to the higher unit weights associated with the harder asphalt. On the basis of these tests, a value of 5.9 percent by dry weight of aggregate was used for specimens containing both the S-2 (40-50 pen) asphalt and S-3 (120-150 pen) asphalt so that comparisons in this instance could be made at constant asphalt content.

Test results for specimens containing Cache Creek gravel and asphalt S-1 are presented in Fig. 3. Based on the conventional mix design criteria, an asphalt content of 4.5 percent was selected for the test beams. This is considerably less than that used in specimens containing the Watsonville granite (5.6-5.9 percent).

Preliminary test results on beam specimens prepared by kneading compaction and sawed with a diamond tipped table saw to final dimensions of 2.0 x 2.7 x 12 in. are shown in Table 3. In this case, as noted in the earlier report, air void content is indicated. For the constant-strain series, approximately 140 beams (exclusive of the S-1 series) have been tested.

In addition to the constant-strain tests, a series of constant-stress fatigue tests were performed on specimens supplied by the Materials and Research Department which had been subjected to cycles of wetting and drying. These beams, originally 3 x 3 x 12 in, were sawed into 1.4 x 1.4 x 12 in. beams. A total of 56 beam specimens were obtained from the 14 bars supplied by the Department. Specific gravity data for each of these beams is presented in a subsequent table also containing the results of fatigue tests.

TEST EQUIPMENT AND PROCEDURES

During this past year, two types of fatigue tests have been conducted: constant-strain and constant-stress (or constant load) tests.

Constant-Strain Fatigue Tests

The constant-strain tests have been described extensively in a previous project report. It should be noted again, however, that the intensity of tensile strain repeatedly applied is monitored by variable resistance bonded wire strain gages nominally 1 in. in length bonded to the lower surface of each of the beams with a thin film of epoxy cement. As will be noted from Figs. 4 and 5 it is necessary to decrease the load during these tests in order to maintain constant strain. The criterion for service life in these tests is defined as that number of repetitions corresponding to a reduction in modulus of rupture to 90 percent of the original unflexed value. This criterion is illustrated in Fig. 6 for tests at 40°F utilizing the specimens containing Watsonville granite and asphalt sample S-2. A summary of the moduli of rupture for unflexed specimens in the series tested this past year is contained in Table 4.

Constant-Stress Fatigue Tests

The constant-stress fatigue tests were performed with equipment developed by Dr. John A. Deacon for his doctoral research on fatigue of asphalt concrete.[3] This equipment is illustrated in Figs. 7 and 8. Fig. 7 contains an overall view of the apparatus and Fig. 8 illustrates details of the load (2) and reaction (1) clamps and the load cylinder (9). This equipment has the capabilities of applying up to five different levels of stress to any one specimen either in repeated small blocks or randomly.

Idealized load vs. time and deflection vs. time curves for this equipment are illustrated in Fig. 10. In this figure it will be noted that load reversal occurs whereas the deflection does not reverse. The return load is used to force the specimen back to its undeflected position after loading.

Essentially a constant moment and hence a constant stress is applied to the center 4 in. of the beam specimen. For the test results reported herein, a constant stress of 100 psi was repeatedly applied at 75°F with a duration of stress of approximately 0.1 sec and a frequency of 100 applications per min. As noted earlier, these tests were performed on specimens supplied by the Materials and Research Department, which had been subjected to cycles of wetting and drying.

To insure that each beam was tested at 100 psi bending stress, preliminary load vs. deflection data were obtained at the 0.1 sec load duration by subjecting each specimen to a few repetitions of a known load and measuring the resulting deflection with a linear variable differential transformer. From the load vs. deflection data, the dynamic stiffness was determined, and the load applied by the apparatus was then adjusted to produce the desired stress of 100 psi. The dynamic stiffness was determined from the relation:

$$\text{Stiffness} = \frac{C_1 P}{I \cdot \Delta} \text{ (psi)}$$

where: P = applied load, lb.

I = moment of inertia, in.^{4}

Δ = center deflection, in.

C_1 = constant depending on apparatus dimension

During the constant-stress tests the load was maintained constant. Periodically, the deflection was measured and the stiffness calculated. Fig. 10 illustrates typical reductions in stiffness of the beam specimens as the load is repeatedly applied.

In these tests, as compared with the constant strain test, the cycles-to-failure represent the number of load applications at which each specimen completely fractured.

FATIGUE DATA

Constant-Strain Tests

Utilizing analyses for various strain levels such as that illustrated in Fig. 6 for specimens containing Watsonville granite and asphalt sample S-2, a series of strain vs. cycles-to-failure relationships were prepared for the project materials and are illustrated in Figs. 11, 12, and 13.

Figure 11 contains the results of tests for specimens containing Watsonville granite and the S-Series asphalts. Also shown in this figure for purposes of comparison are the results of tests on specimens with the same aggregate and the G and E asphalts reported earlier. It will be noted that the results for the asphalts with different penetrations differ from those reported previously, in that the initial hardness of the asphalt appears to influence the strain vs. cycles-to-failure results at 40°F. For the S-Series materials it would appear that the softer the asphalt, the longer the fatigue life at a given strain level. These results, however, are related to specimen stiffness. In general, the less stiff the specimen, the greater the number of load applications to failure in constant-strain tests. This statement is substantiated at least qualitatively by the data presented in Table 5. In this table the load to cause a bending strain of 350×10^{-6} in. per in. at 40°F is summarized for the test series containing Watsonville granite. In the case of asphalt samples S-1 and S-3, this stiffness concept would not appear to be completely correct. However, if one examines the average air void content for each of these series it will be noted that on the average the air void content for the specimens containing asphalt S-3 is 1.2 percent less than that for specimens containing asphalt S-1. This would indicate an increase in density which probably has resulted in slightly stiffer specimens for the S-3 asphalts. (In addition the effect of air void content in terms of longer service life associated with lower air void content is present in this comparison. See Fig. 14 for data substantiating this from another study.[4])

Also of significance is the comparison between the S-series and G-series asphalts. Specimens containing asphalt S-2 (40-50 pen.) are considerably stiffer than those with asphalt G (85-100 pen.), e.g., see Table 5. However, the strain vs. cycles-to-failure relationships for the S-2 series lies to the right of that for the G-series. Thus asphalt source would appear to have a significant effect on fatigue behavior.

Strain vs. cycles-to-failure results for specimens containing the Cache Creek aggregate and asphalt S-1 are presented in Fig. 12. Essentially the same trends are noted for these specimens as were obtained for the specimens containing Watsonville granite. It will be noted in Table 5 that specimens containing asphalt S-1 and Cache Creek gravel are stiffer than those containing the same asphalt and Watsonville granite. On a relative density basis, the specimens with the Cache Creek aggregate are considerably more compact than those with the Watsonville granite (3.6 percent vs. 5.5 percent air voids). In spite of this there is a difference of 4 to 5 times in fatigue life at 40°F. This comparison is illustrated in Fig. 13. A similar relationship also appears to exist at the higher temperature. Additional data, however, are now being developed in this range. The effect of asphalt content would appear to be one of the prime factors in the increased life for specimens containing the Watsonville granite. For these specimens a greater amount of asphalt can be tolerated at a specific level of stability as compared to specimens utilizing the uncrushed gravel. It is interesting to conjecture what the relative effect would be were both of the materials compared at the same relative density. For example, comparing the two mixes at a void content of 5.5 percent, the curve for specimens containing Watsonville granite would remain as indicated. The curve for the gravel would, in all probability, be displaced to the left thus showing a greater disparity in cycles to failure at particular strain level.

Constant-Stress Tests

Results of the constant-stress fatigue tests on a series of specimens subjected to cycles of wetting and drying are summarized in Table 6. As noted previously these specimens were supplied by the Materials and Research Department as 3 in. x 3 in. x 12 in. bars, which in turn were sawed to specimens approximately 1.5 in. x 1.5 in. in cross-sectional area.

Examination of the data in Table 6 indicates that for this aggregate the cycles of wetting and drying to which the specimens were subjected had no apparent influence on fatigue life, at least at the one stress level, 100 psi, investigated.

Of significance, however, is the influence of asphalt content. It will be noted in Table 6 that specimens containing the same aggregate with two levels of asphalt content were tested. In general the specimens prepared at the higher asphalt content exhibited a considerably greater fatigue life (approximately 3 times as great based on overall averages).

Also of interest, though not as well defined, is the influence of density. The lower portion of each bar (specimens 3 and 4) appear generally to be at a lower density and in a number of cases exhibit a lower fatigue life.

While not a part of the fatigue data, constant stress tests to determine dynamic stiffness were performed on specimens 1.5 in. x 1.5 in. x 15 in. in length prepared using the S-Series asphalts and both aggregates. Stiffness measured both at 40°F and 75°F at a stress of 100 psi and a time of loading of 0.1 sec are summarized in Table 7. As was previously noted for the constant-strain tests, the stiffness of the Cache Creek material is either higher or of the same order as the Watsonville granite specimens when asphalt of the same hardness is utilized. Also shown is the influence of hardness of the asphalt for Watsonville granite specimens. In this instance the effect of hardness of asphalt is less marked at the lower temperature (when comparisons are made at the same degree of densification).

DISCUSSION

In the previous section it was indicated that asphalt stiffness influences the results of both constant-stress and constant-strain fatigue tests. Also it has been indicated that mixture stiffness is related to the hardness of the asphalt contained in the mixture. Thus to begin developing a tie-in between laboratory measured performance and field conditions, tests were performed on asphalts extracted from a series of laboratory fabricated beam specimens by the Materials and Research Department staff. The results of viscosity* tests on the extracted asphalts are illustrated in Table 8. These results together with original viscosity tests,* and tests* performed on the asphalt after the Rolling Thin Film Test (State of California Test Method No. 346) are presented in Figs. 15, 16 and 17 for asphalts G, E, and those in the S-Series, respectively.

From this data it would appear that the mixtures prepared in the laboratory contain asphalts which have not been hardened to the extent that might occur in the hot-mixing process since the viscosity temperature curves lie below the viscosity results obtained after the Rolling Thin Film Test.[5] That some hardening is apparent, however, is indicated by the location of the viscosity curve for recovered material as compared to that for the original asphalt.

Thus the stiffness of the mixtures tested may be less than that which could be developed in new pavements following construction. The laboratory curves for constant-strain tests in all probability lie somewhat to the right of curves which might be obtained for samples from actual pavements containing the same materials.

Viscosity measurements on asphalts recovered from in-service pavements should also prove useful in analyses of actual pavement behavior. The Royal Dutch Shell research group has developed a procedure whereby from a knowledge of the penetration and softening point of the asphalt in the mixture, a measurement of stiffness of the mixture at a particular time of loading and temperature can be determined. The nomograph shown in Fig. 18 was developed by Van der Poel[6] as a means for determining the stiffness of asphalt utilizing its penetration at 77°F and ring and ball softening point temperature.

Recently Heukelom and Klomp[7] have presented a convenient means for determining the stiffness of the mixture provided the volume concentration (C_v) of the aggregate and the stiffness of the asphalt are known. These relationships are illustrated in Fig. 19. The curves are based on the equation

* Viscosity measured where appropriate at 0.05 sec^{-1}.

$$\frac{S_{mix*}}{S_{bit.*}} = \left(1 + \frac{2.5}{n} \cdot \frac{C_v}{1-C_v}\right)^n$$

where: $n = 0.83 \log \frac{4 \cdot 10^5}{S_{bit*}}$

and are applicable to well compacted mixtures with about 3 percent air voids and C_v values ranging from 0.7 to 0.9.

Thus it is convenient to develop at least an approximation to the stiffness characteristics of an asphalt concrete mixture, provided the penetration and ring and ball softening point tests are performed on the recovered material. In the analysis of various field projects where asphalt is recovered from the mixture, it would appear convenient to measure the above noted test properties so that stiffness data would be available. This is particularly important if it is desired to interpret field measurements and performance within the framework of theory.

That this approach is not unreasonable is substantiated by results obtained by Deacon[3] for specimens of Watsonville granite and Standard Oil Company 85-100 pen. asphalt cement. The penetration at 77°F and ring and ball softening point of the recovered asphalt were 40 dmm and 121°F, respectively. Utilizing Figs. 18 and 19, the stiffness was computed to be 2 to 2.5×10^5 psi corresponding to a time of loading of 0.1 sec. and a temperature of 75°F. The actual measured stiffness was in the range 2.5 to 2.6×10^5 psi depending upon the stress level used to measure the deflection and with a standard deviation in this range of the order of 30,000 psi. The data presented in Table 8, unfortunately, did not include the ring and ball temperature; therefore the results in Table 7 cannot be compared with the approximate procedure.

It should be emphasized that the above suggestions are not made to supplant detailed testing of individual pavement sections in any field program. However, as indicated subsequently in the section on recommendations for investigations for the year 64-65, only a few pavements can be investigated in detail. Assuming, however, that tests will be performed on cores from the majority of sections by the Materials and Research Department, the additional data discussed above could be attained inexpensively and could have potential in analysis of performance.

During the past few years there have been suggestions that cumulative damage concepts could be utilized to determine the effects of wheel loads of different intensities

* Stiffnesses measured in kg per sq cm.

such as those occurring on an actual pavement.[8,9] Some recent work by Deacon[3] has indicated that a modification of the simple linear summation of cycle ratios for different stress (or strain) intensities may be a reasonable approximation to incorporate the effects of various wheel loads. Already the linear summation of cycle ratios concept is embodied in a procedure with some theoretical basis prepared by the Shell Oil Company.[10] Thus with actual fatigue data such as that which is being developed coupled with the periodic stiffness and fatigue measurements, predictions of service lives on actual pavements might be obtained.

From the data developed in this investigation thus far, it would appear that each mix exhibits its own unique fatigue behavior and that designs should be accomplished for each mixture. It is possible, however, for well designed mixtures in use in heavy duty highways, that a pattern of behavior such as that developed by Nijboer and Heukelom[7] and illustrated in Fig. 20 might exist at least to a first approximation. If this is the case, then mixture design to incorporate fatigue behavior could be simplified.

To illustrate this point, the data obtained from two investigations, one by Deacon[3] and one by Hicks,[11] using constant-stress tests with the equipment illustrated in Fig. 7 and a mixture composed of Watsonville granite and 85-100 penetration asphalt are summarized in Table 9. The stiffness values listed for the various temperatures are average values since it has been shown[3] that stiffness is stress dependent. It should also be noted that while the aggregate and asphalt were the same in both the Deacon and Hicks investigations, the asphalt did not change its characteristics to the same extent in Hicks' investigation as compared to Deacon's study (based on penetration and ring and ball softening point tests on the recovered asphalt).

The data in Table 9 have been plotted in Fig. 21 using the same procedure suggested in Fig. 20. While only a few data points are available, it is not too difficult to visualize the same trends as presented in Fig. 20. The lines corresponding to the different repetitions are shown by dashed lines, however, to emphasize that the trends are tentative since only limited data have been utilized.

RECOMMENDATIONS FOR RESEARCH FOR 1964-65

As recommended last year, one of the objectives should be the initiation of a program to relate field performance to laboratory measured fatigue characteristics. To this end the Materials and Research Department has submitted 7 projects for consideration. The various projects are summarized in Table 10. These projects are a portion of a total of 25 projects for which deflection and performance criteria are being periodically evaluated. In terms of material available, range of pavement types, deflection, and traffic the following three projects are initially recommended for laboratory and subsequent field studies.

1. V-Mon-2-C
2. V-SLO-56-C,D
3. IV-Nap-8,49-Nap A,D

In the laboratory, it is proposed that constant-stress tests be performed utilizing the equipment illustrated in Fig. 7 to define the fatigue characteristics of the mixtures. It is recommended that the tests be conducted at two temperatures, one in the vicinity of 40°F and the other near 70°F. This would appear to give a reasonable indication of the effects of temperature in a range where fatigue cracking may be a factor in all three localities.

As was noted in the previous report, neither constant-stress nor constant-strain tests may strictly be applicable in defining conditions which occur in the pavement. Thus, some constant-strain tests will also be performed in order to define the range in behavior. Some clarification should be obtained in the near future on the comparison between constant-stress and constant-strain tests and under what conditions (at least approximately) one or the other test procedures are applicable from another investigation currently under way.

In addition to these fatigue tests on laboratory prepared specimens, additional tests should be performed on specimens cut from the pavement as soon as practicable and at periodic intervals in the future to define the change in fatigue characteristics with time and the influence of traffic on these characteristics.

It is hoped that tests can be performed to give a measure of the resilience characteristics of the untreated granular materials and fine-grained soils as well as the treated materials underlying the asphalt concrete sections. Some form of field vibratory testing as well as the field deflection measurements should be considered. These measurements should assist greatly in any analysis which will be developed.

In addition to the above program some tests must still be completed on Cache Creek gravel and the Watsonville granite with the S-Series asphalts.

SUMMARY AND CONCLUSIONS

In the investigation to date an attempt has been made to define the influence of asphalt type and hardness and aggregate type on the behavior of asphalt concrete in constant-strain amplitude fatigue tests. These data suggest:

1. Asphalt type would appear to affect the results, e.g., comparison of the results of specimens prepared with asphalt G and S-2 at 40°F.

2. For asphalts from the same source, the initial hardness may or may not affect the strain vs. cycles-to-failure relationship at a particular temperature depending on the stiffness which the asphalt gives to the mixture at the test temperature. In the case of the asphalts (40-50 and 85-100 pen.) used on the Contra Costa County Shell Ave. Test Road, both materials appeared to impart about the same stiffness to the mixture at the test temperatures and thus no difference in behavior was observed. On the other hand, asphalt S-2 (40-50 pen.) gives a greater mixture stiffness at 40°F than does S-1 (85-100 pen.) and correspondingly there is a difference in the strain-cycles-to-failure relationship at this temperature.

3. Aggregate type would appear to influence the results of constant-strain tests. However, for the materials investigated, it is difficult to separate the effect of asphalt content from that of aggregate type. In the case of the Cache Creek gravel, the mixture could only tolerate 4.5 percent asphalt whereas the Watsonville granite, with the same asphalt type, contained 5.9 percent asphalt.

In addition to the constant-strain tests, constant-stress tests have been performed. The results of these tests serve to emphasize the importance of as much asphalt as possible for good fatigue behavior and also serve to indicate the importance of proper compaction of mixtures in that higher densities for a given design appear to be associated with better performance.

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TABLE 1 – SPECIFIC GRAVITY DATA–CACHE CREEK GRAVEL
(ASTM C-127, 128)

	Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
Coarse Aggregate	2.64	2.74	1.4
Fine Aggregate	2.59	2.63	0.6

TABLE 2 – COMPARATIVE TEST PROPERTIES, NEW AND ORIGINAL SAMPLES – S-SERIES

Asphalt	Pen. 77°F 100 gr, 5 sec	Pen. 39.2°F 200 gr, 60 sec	Pen. Ratio	Viscosity 140°F x 10^5 Cs	Viscosity 275°F Cs
40-50 (new)	42	10	23.8	3.37	393.3
40-50 (original)	39	-	24.0	3.91	492.0
85-100 (new)	93	30	32.3	1.32	303.0
85-100 (original)	96	-	32.0	1.53	348.0
120-150 (new)	129	39	30.2	0.764	227.0
120-150 (original)	119	-	32.0	0.902	276.0

TABLE 3 — PRELIMINARY TEST RESULTS ON LABORATORY PREPARED SPECIMENS

Test Series		Test Temperature (°F)	Total No. of Specimens	Percent Air Voids		
Aggregate	Asphalt			Mean	Standard Deviation	Coef. of Variation
Watsonville	S-1	40	26	5.5	0.4	7.3
Watsonville	S-2	40	49	5.1	0.8	15.2
Watsonville	S-3	40	50	4.3	0.5	11.1
Cache Creek	S-1	40	33	3.6	0.5	12.6

TABLE 4 — MODULUS OF RUPTURE OF UNFLEXED SPECIMENS

Test Series		Test Temperature (°F)	Total No. of Specimens	Modulus - psi		
Aggregate	Asphalt			Mean	Standard Deviation	Coef. of Variation
Watsonville	S-1	40	10	690	67.0	9.7
Watsonville	S-2	40	18	984	111	11.3
Watsonville	S-3	40	17	572	45.0	7.9
Cache Creek	S-1	40	12	807	106	13.1

TABLE 5 — LOAD TO PRODUCE STRAIN AMPLITUDE OF 350×10^{-6} IN. PER IN. AT 40°F

Test Series		Asphalt Content (Percent)	Percent Air Voids (Avg.)	Load		
				Start of Test		At Estimated Fatigue Life
Aggregate	Asphalt			Avg. (lb)	Range (lb)	Avg. (lb)
Watsonville	E	5.8	5.1	295	270-345	205
Watsonville	G	5.6	5.4	490	365-540	380
Watsonville	S-1	5.9	5.5	387	310-460	236
Watsonville	S-2	5.9	5.1	807	710-855	516
Watsonville	S-3	5.9	4.3	417	360-470	310
Cache Creek	S-1	4.5	3.6	575	440-660	450

TABLE 6 — SUMMARY OF CONSTANT STRESS FATIGUE TESTS PERFORMED AT 75°F

Series Number		4.5% Asphalt Content			5.5% Asphalt Content		
		Specific Gravity	Cycles to Failure - N_f	Ave. N_f	Specific Gravity	Cycles to Failure - N_f	Ave. N_f
1 - Control	1	2.40	14,070	4,400	2.46	18,990	12,600
	2	2.44	1,760		2.46	7,770	
	3	2.34	952		2.40	13,570	
	4	2.36	842		2.41	10,040	
2 - 1 Cycle	1	2.45	6,400	4,970	2.46	18,350	21,000
	2	2.43	1,690		2.47	25,850	
	3	2.36	3,650		2.40	16,550	
	4	2.37	8,100		2.39	23,165	
3 - Control	1	2.45	5,000	6,830	2.45	42,500	17,400
	2	2.50	13,000		2.48	12,430	
	3	2.37	2,300		2.41	8,600	
	4	2.36	6,970		2.40	5,930	
4 - 3 Cycles	1	2.45	13,000	7,000	2.45	21,600	15,000
	2	2.45	5,000		2.44	30,000	
	3	2.37	2,086		2.40	4,480	
	4	2.36	7,860		2.37	3,780	
5 - Control	1	2.43	2,650	4,700	2.46	35,000	21,600
	2	2.36	5,500		2.46	31,500	
	3	2.43	6,000		2.40	8,200	
	4	2.38	–		2.41	11,500	
6 - 6 Cycles	1	2.45	15,990	9,100	2.47	66,800	25,000
	2	2.43	14,800		2.47	5,900	
	3	2.38	4,245		2.38	14,350	
	4	2.38	1,315		2.45	13,120	
7 - Control	1	2.42	960	2,590	2.47	42,900	18,900
	2	2.40	4,255		2.48	17,220	
	3	2.36	4,330		2.38	7,325	
	4	2.38	810		2.43	8,260	

TABLE 7 — DYNAMIC STIFFNESS MEASUREMENTS

Aggregate	Asphalt Grade[2]	Asphalt Content (Percent)	Tests at 40°F: Percent Air Voids	Tests at 40°F: Dynamic Stiffness[1] (psi)	Tests at 75°F: Percent Air Voids	Tests at 75°F: Dynamic Stiffness[1] (psi)
Watsonville	85-100	5.9	3.6	52.6×10^4	3.7	10.6×10^4
			4.5	46.7×10^4	4.6	7.7×10^4
			3.0	65.5×10^4	5.1	7.3×10^4
	40-50	5.9	3.5	52.2×10^4	3.9	30.0×10^4
			1.8	114×10^4	3.9	25.9×10^4
			3.4	65.9×10^4	3.5	24.9×10^4
Cache Creek	85-100		3.6	52.2×10^4	4.0	14.3×10^4
			4.0	47.5×10^4	3.2	14.7×10^4

(1) Time of loading — 0.1 sec, stress of 100 psi.
(2) Asphalt type — S-Series.

TABLE 8 — TEST RESULTS ON ASPHALTS RECOVERED FROM BEAM SPECIMENS*

Source and Grade	Asphalt E 85-100	Asphalt S-2 40-50	Asphalt S-1 85-100	Asphalt G 85-100
Penetration at 77°F	50	35	43	46
40°F				
Viscosity at 0.05 sec^{-1} megapoise	300.0	695.0	278.0	850.0
Viscosity at 0.001 sec^{-1} megapoise	1330.0	3700.0	2150.0	2900.0
Shear Susceptibility	.38	.43	.52	.31
60°F				
Viscosity at 0.05 sec^{-1} megapoise	29.7	120.0	51.0	64.5
Viscosity at 0.001 sec^{-1} megapoise	105.0	242.0	152.0	81.5
Shear Susceptibility	.32	.18	.28	.06
77°F				
Viscosity at 0.05 sec^{-1} megapoise	5.05	11.0	6.7	3.9
Viscosity at 0.001 sec^{-1} megapoise	7.1	14.7	10.6	4.8
Shear Susceptibility	.09	.08	.11	.05
Microductility mm	36	63	27	98
100°F				
Viscosity at 0.05 sec^{-1} megapoise	.30	.62	.45	.177
Viscosity at 0.001 sec^{-1} megapoise	.40	.62	.52	.20
Shear Susceptibility	.08	.00	.04	.03
140°F				
Viscosity	3969.9 poise	7509.2 poise	6827.2 poise	2667.1 poise

* Tests performed by Materials and Research Department, California Division of Highways.

TABLE 9 – TEST RESULTS–CONSTANT STRESS TESTS
WATSONVILLE GRANITE AND 85-100 PEN. ASPHALT CEMENT

Temperature	Avg. Stiffness (psi)	Stress (psi)	Initial Tensile Strain (in./in.)	Repetitions to Failure (N)
75°F(1)	250,000	507	2.02×10^{-3}	10
		204	8.16×10^{-4}	10^3
		129	5.15×10^{-4}	10^4
		83	3.32×10^{-4}	10^5
68°F(2)	180,000	190	1.05×10^{-3}	10^3
		97	5.40×10^{-4}	10^4
		50	2.78×10^{-4}	10^5
40°F(2)	1,000,000	470	4.70×10^{-4}	10^3
		360	3.60×10^{-4}	10^4
		277	2.77×10^{-4}	10^5

(1) Data obtained by J. A. Deacon.
(2) Data obtained by R. G. Hicks.

TABLE 10 — DEFLECTION—TRAFFIC INDEX STUDY PROJECTS

Project	Location	Structural Design	Design Traffic Index	Materials	Comments
V-Mon-2-C	Gonzales Bypass US 101	3.6" A.C. 3" A.C. 6" A.B. 15" A.S.	1961 - 1971 9.1 32.2×10^6 EWL	Surface course: 85-100 pen a.c. Union 2 gal available Base course: 85-100 pen. a.c. Shell 1/2 gal available	Deflections to 0.015" in Feb. 1964
V-Mon-2-G	US 101 Near San Lucas	3" A.C. 8" CTB, Class B 9" A.S.	1963 - 1973 8.3 13.8×10^6 EWL	85-100 pen. a.c. Douglas, Santa Maria 1/2 gal available	Low deflections in March 1964 0.002 to 0.007"
V-SLO-56-C, D	US 1 Morrow Bay to Cayucos	2.5" A.C. 8" A.B. 12" A.S.	1960 - 1970 6.8 2.4×10^6 EWL	85-100 pen. a.c. Union, Santa Maria 1-1/2 gal available	Some deflections exceeding 0.030" in March 1964
V-SB-150 SB, A	Santa Barbara State Highway 150	2.5" A.C. 6" A.B. 8" A.S.	1958 - 1968 6.2 10^6 EWL and 4.8 10^5 EWL	85-100 pen. a.c. Seaside, Ventura 1/2 gal available	
VI-Kin-10-B	State Route 198 (10) Near Lemoore	3" A.C. 6" CTB, Class A 8" A.S. 4" I.B. (min.)	1960 - 1970 7.9 9.31×10^6 EWL	85-100 pen. a.c. Douglas, Santa Maria 2 gals available	Deflections less than 0.010" in March 1964
VI-Kin, Tul-135-B, A	Corcoran Bypass	3" A.C. 6" CTB, Class B 12" A.B. - in some sections	1963 - 1973 7.9 9×10^6 EWL	85-100 pen. a.c. Standard Bakersfield 1 gal available	Deflections up to 0.020" in May 1964
IV-Nap-8, 49-Nap, A, D	Napa State Highways 8, 49	3" A.C. 4" CTB, Class A 4" CTB, Class B 6" A.S. (Type 1) 12" A.S. (Type 2)	1962 - 1972 8.2	120-150 pen. a.c. Shell, Martinez Aggregate-Basalt Rock Co.	Some deflections in excess of 0.020" in Feb. 1964

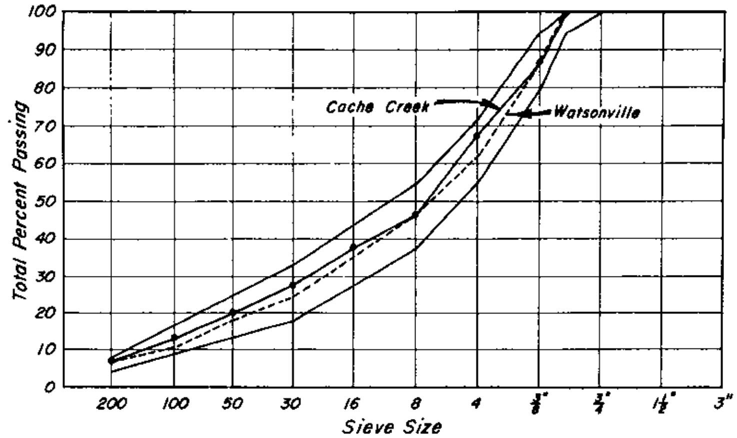


Fig. 1 — Grading curves, Watsonville granite and Cache Creek gravel.

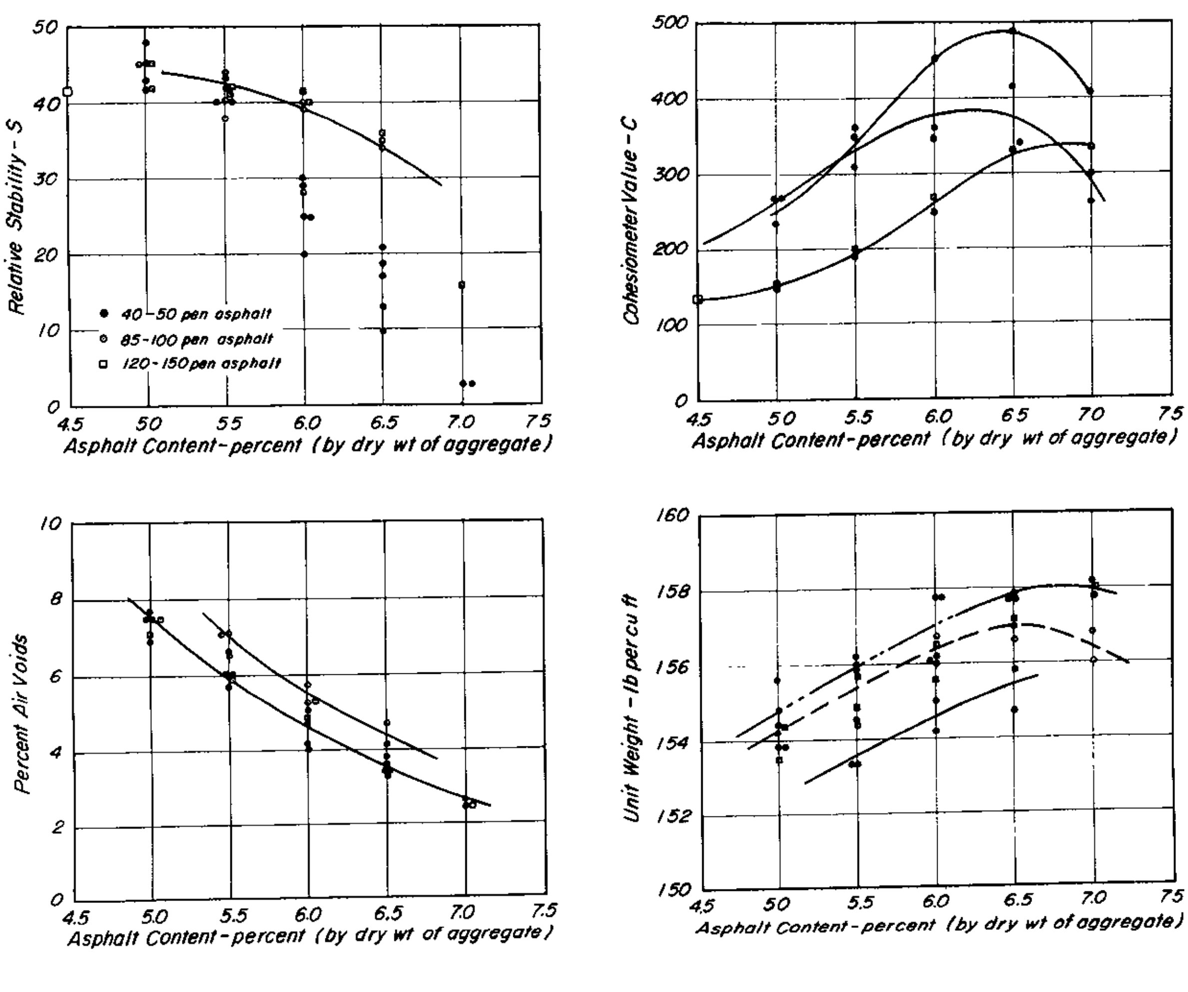


Fig. 2 - Mix design data, Watsonville granite and S-series asphalt.

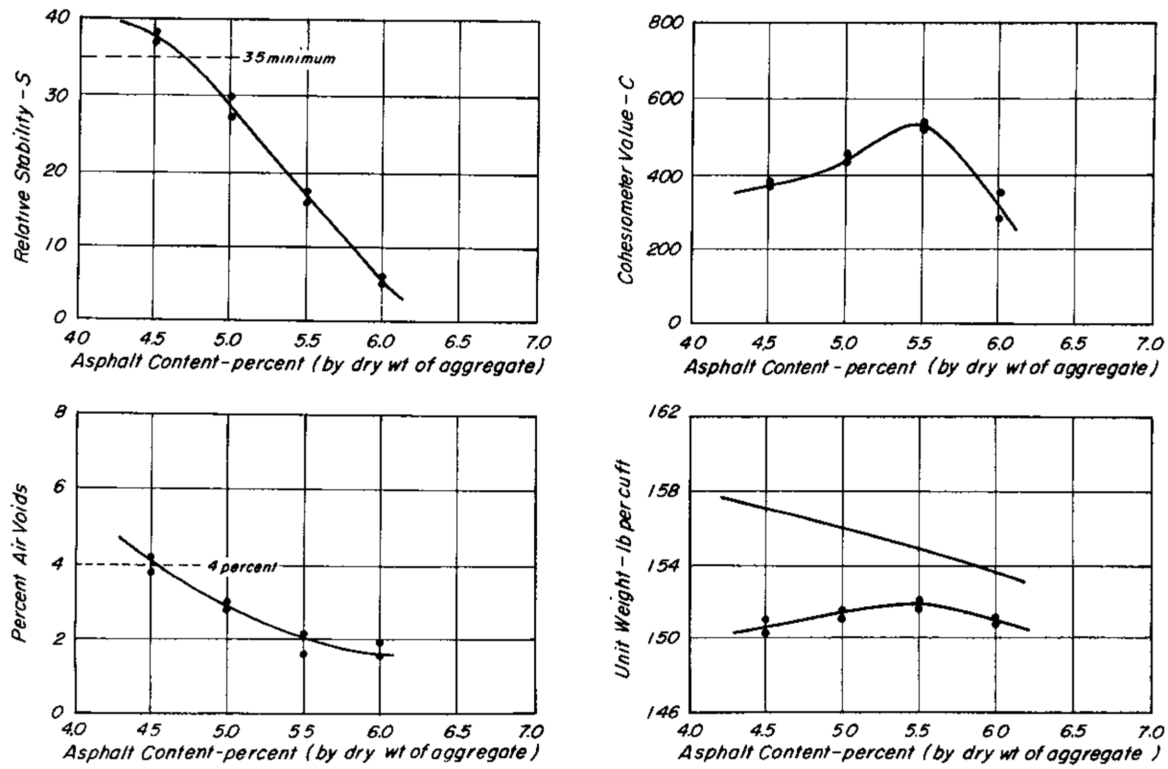


Fig. 3 - Mix design data, Cache Creek aggregate and asphalt S-1.

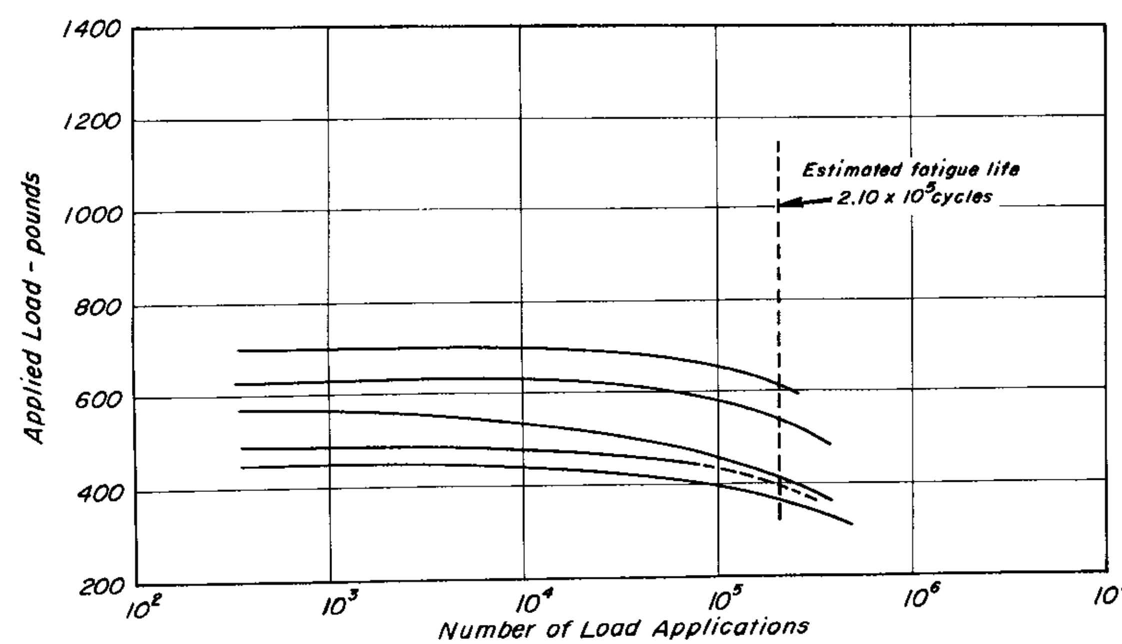


Fig. 4 - Load required to produce 250 x 10^{-6} in. per in. strain vs number of load applications for specimens containing Watsonville granite and asphalt S-2 at 40°F.

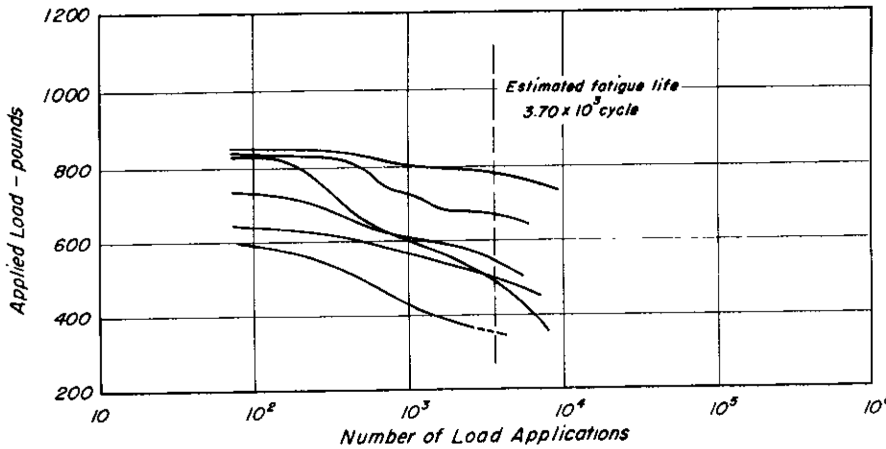


Fig. 5 - Load required to produce 450 x 10^{-6} in. per in. strain vs. number of load applications for specimens containing Cache Creek aggregate and asphalts S-1 at 40°F.

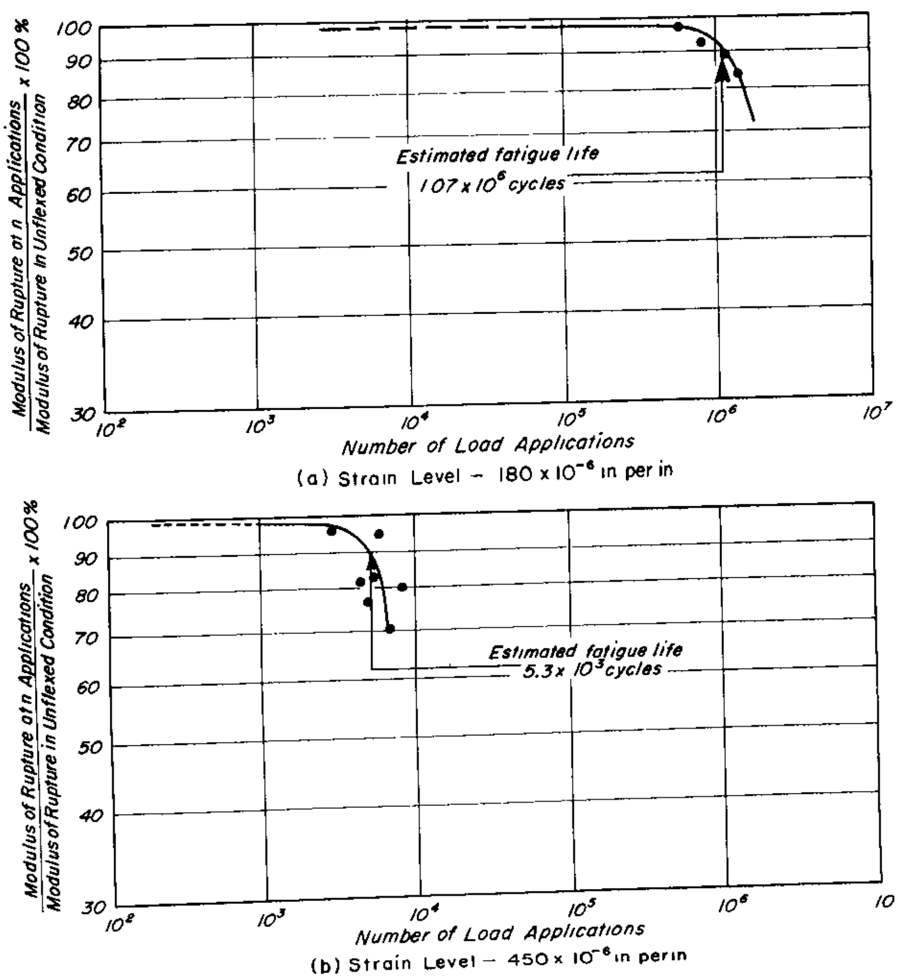
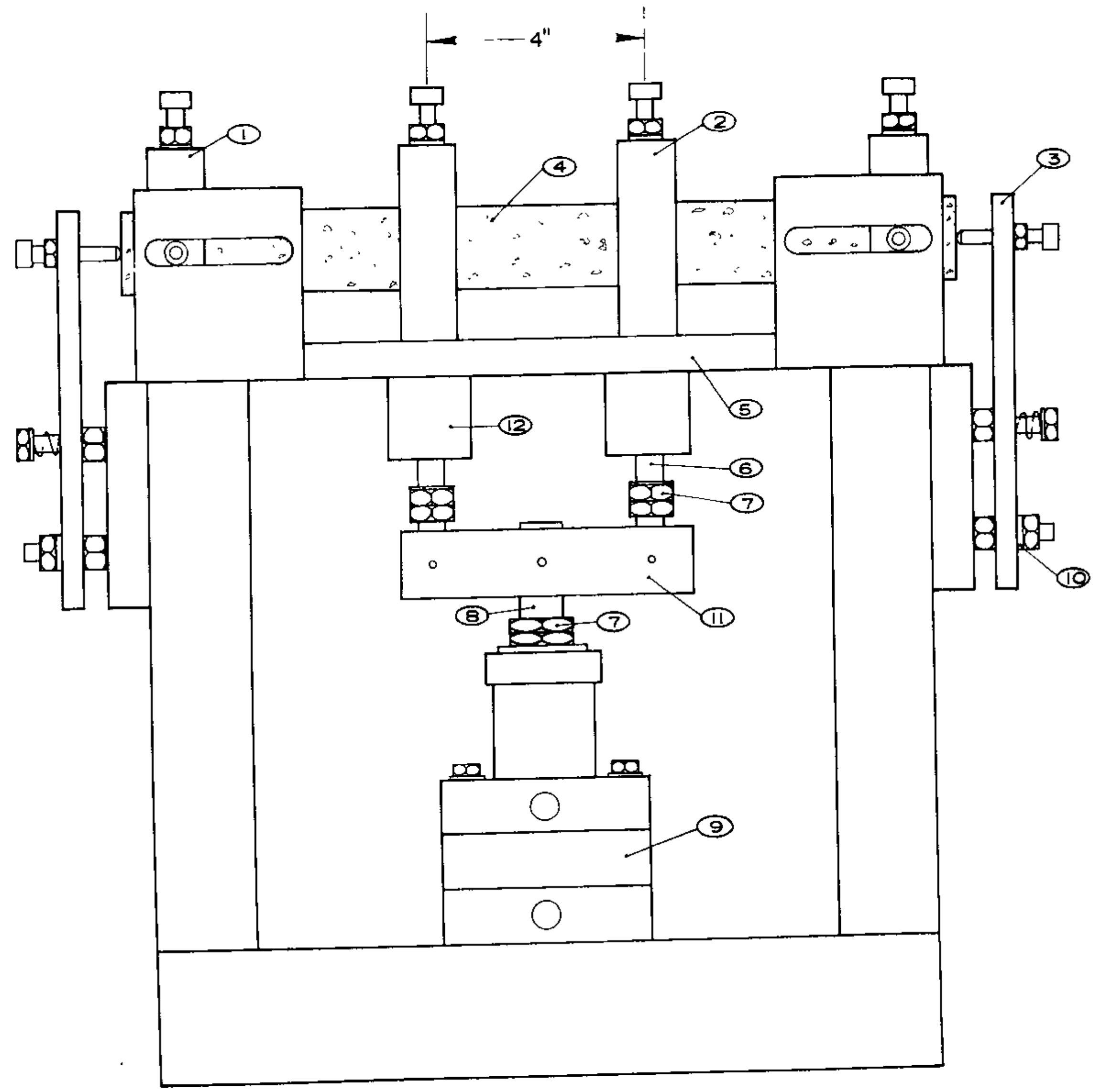


Fig. 6 - Modulus of rupture vs. number of load applications for specimens prepared with Watsonville granite and asphalt sample S-2 at 40°F.



Key:

1. Reaction clamp
2. Load clamp
3. Restrainer
4. Specimen
5. Base plate
6. Loading rod
7. Stop nut
8. Piston rod
9. Double-acting, Bellofram cylinder
10. Rubber washer
11. Load bar
12. Thompson ball bushing

Fig. 7 - Repeated-flexure apparatus.

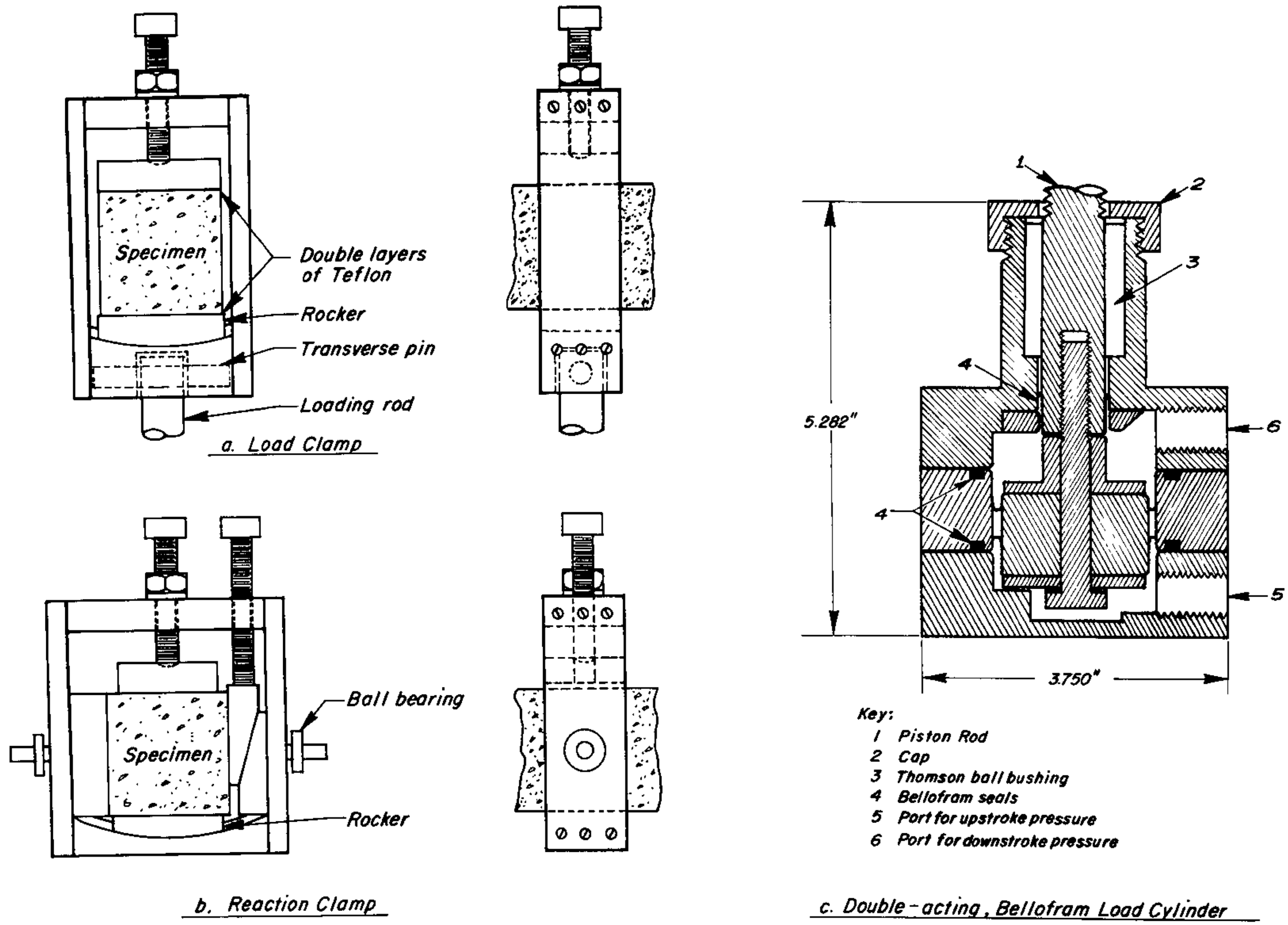


Fig. 8 - Details of components of repeated-flexure apparatus.

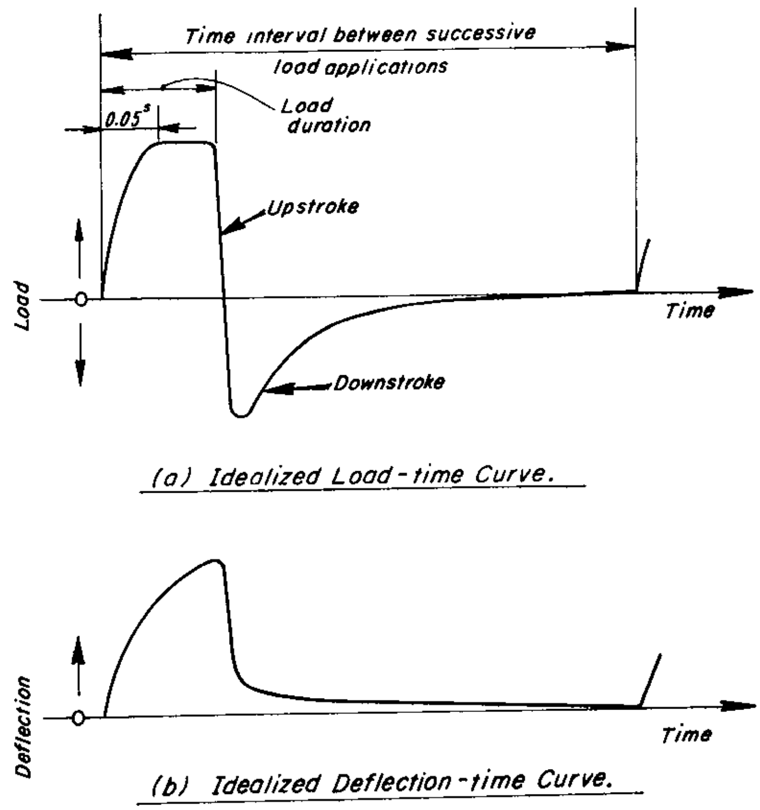


Fig. 9 - Load vs. time and deflection vs. time relationships for constant-stress test equipment.

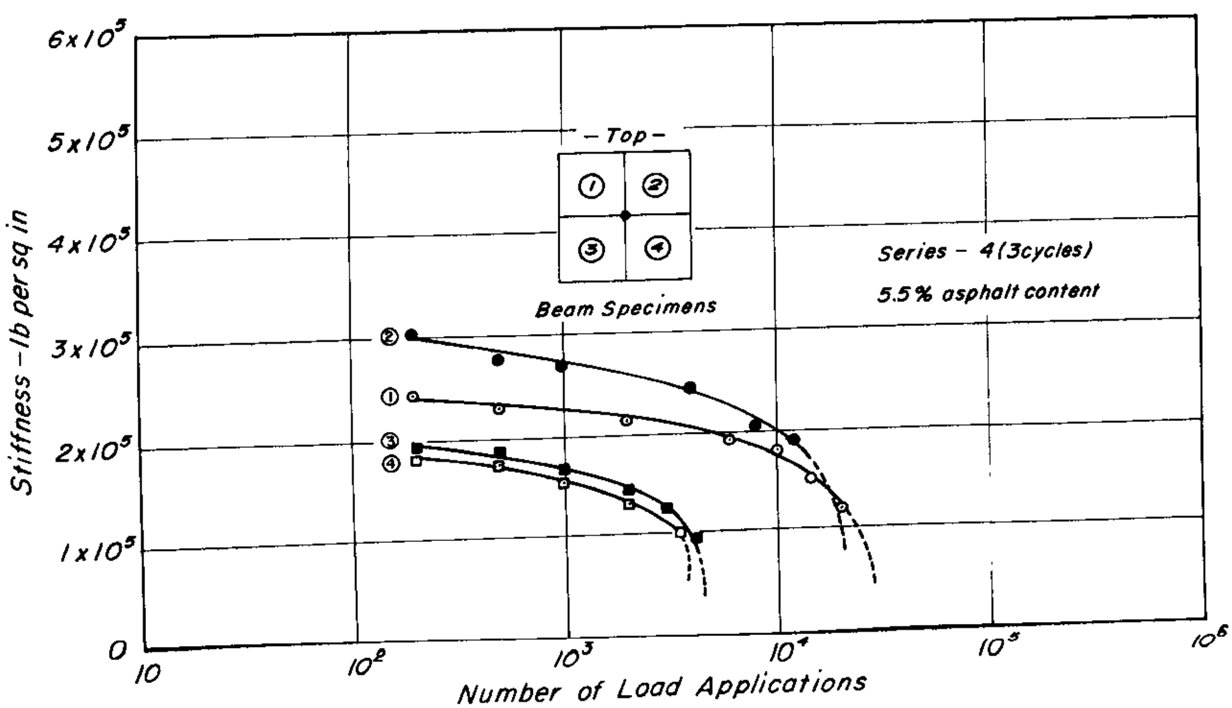


Fig. 10 - Typical reduction in stiffness vs. number of load applications applications - constant stress test, 75° F.

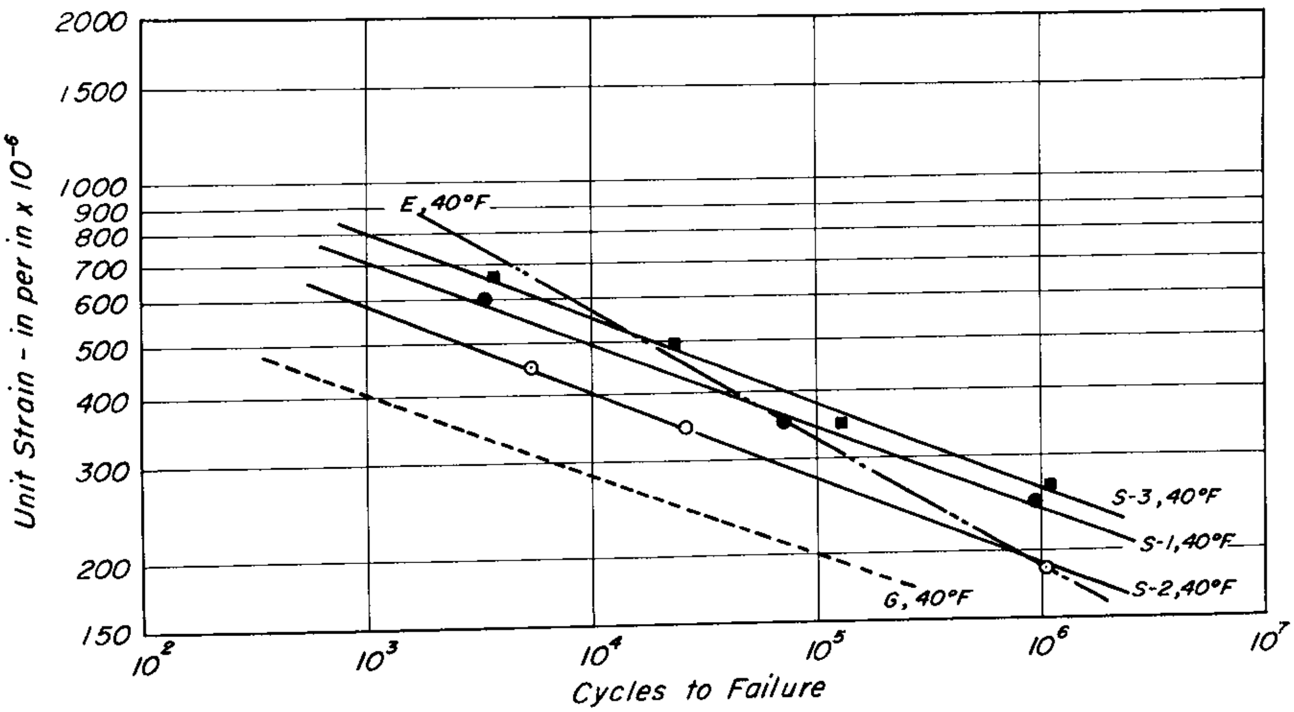


Fig. 11 - Results of constant strain amplitude fatigue tests at 40°F on specimens containing Watsonville granite and S-series asphalts.

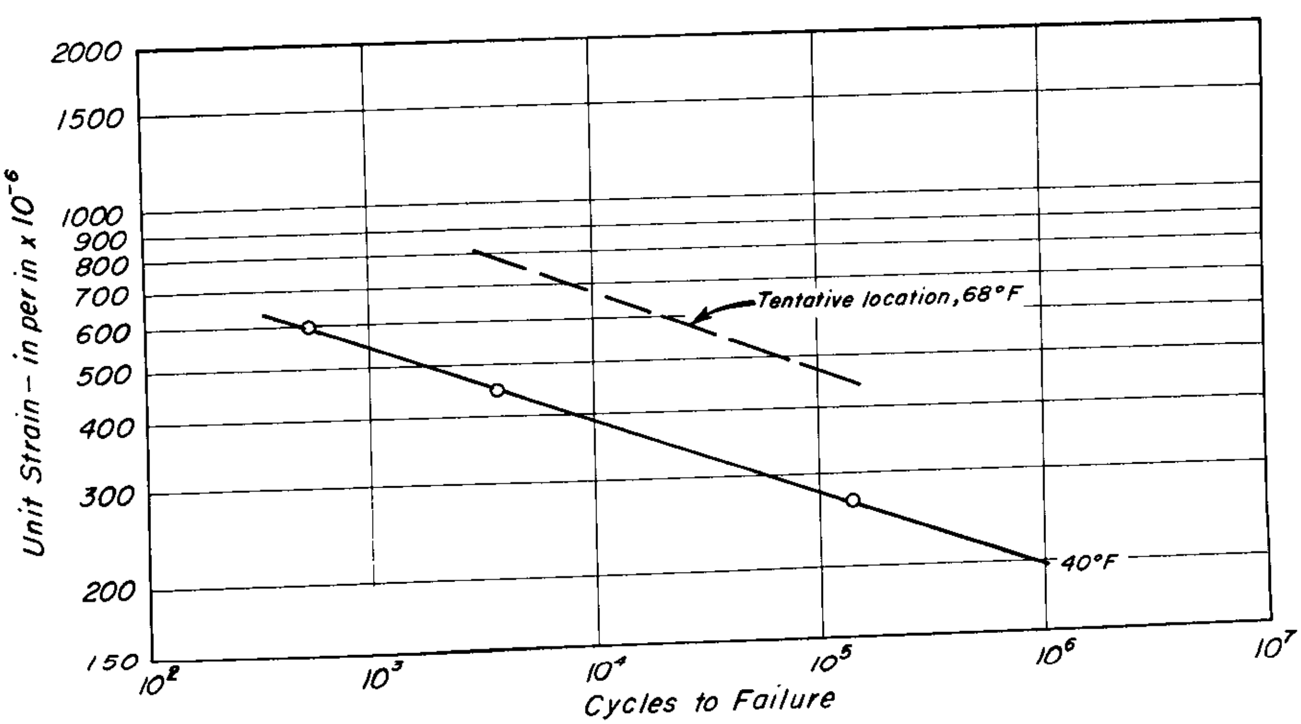


Fig. 12 - Results of constant strain amplitude fatigue tests at 40°F and 68°F on specimens containing Cache Creek aggregate and asphalt S-1.

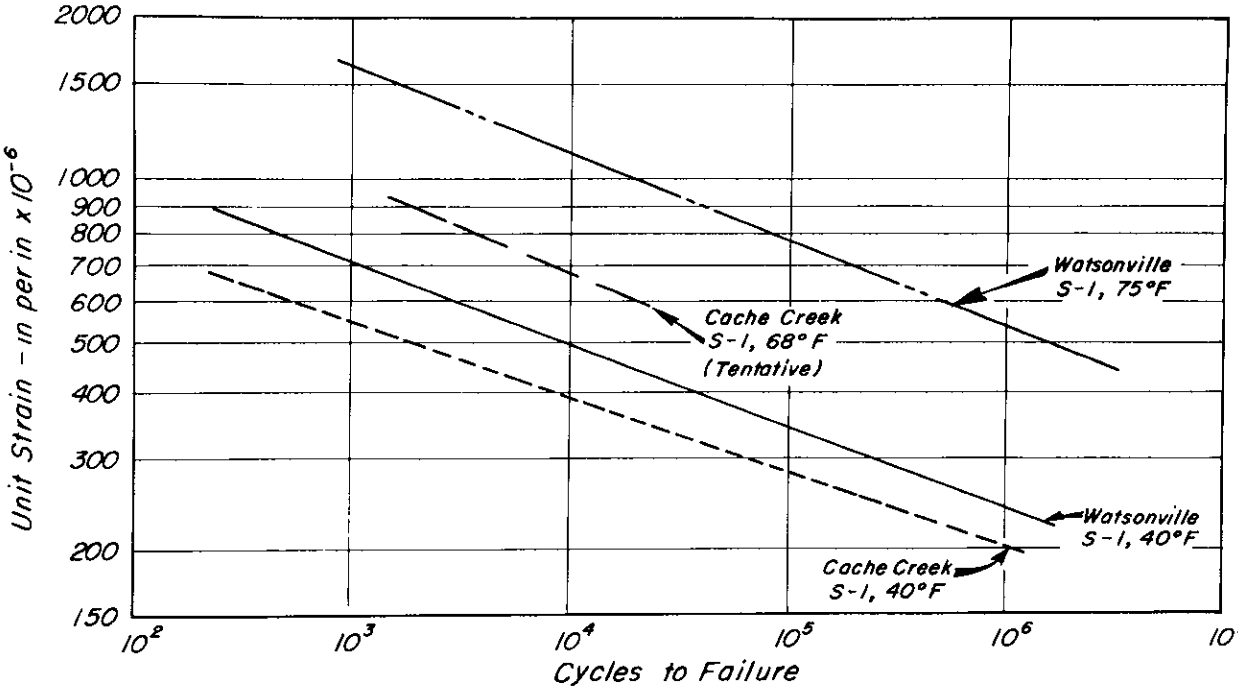


Fig. 13 - Influence of temperature and aggregate type on results of constant strain amplitude fatigue tests.

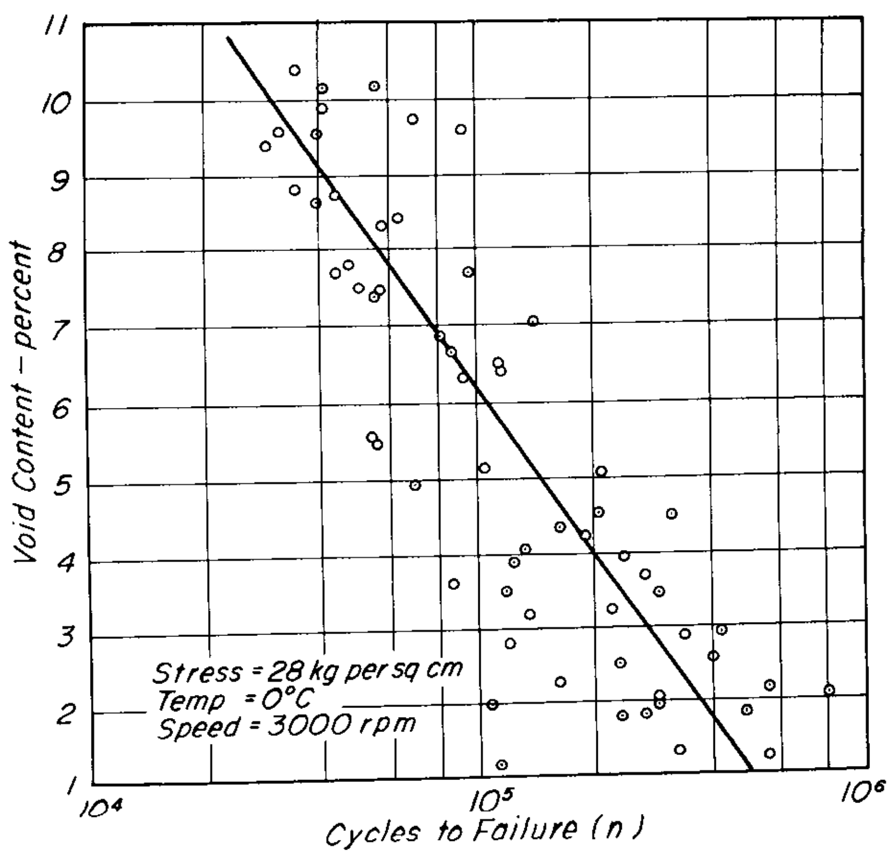


Fig. 14 - The effect of voids content on fatigue life. (After Saal and Pell.)

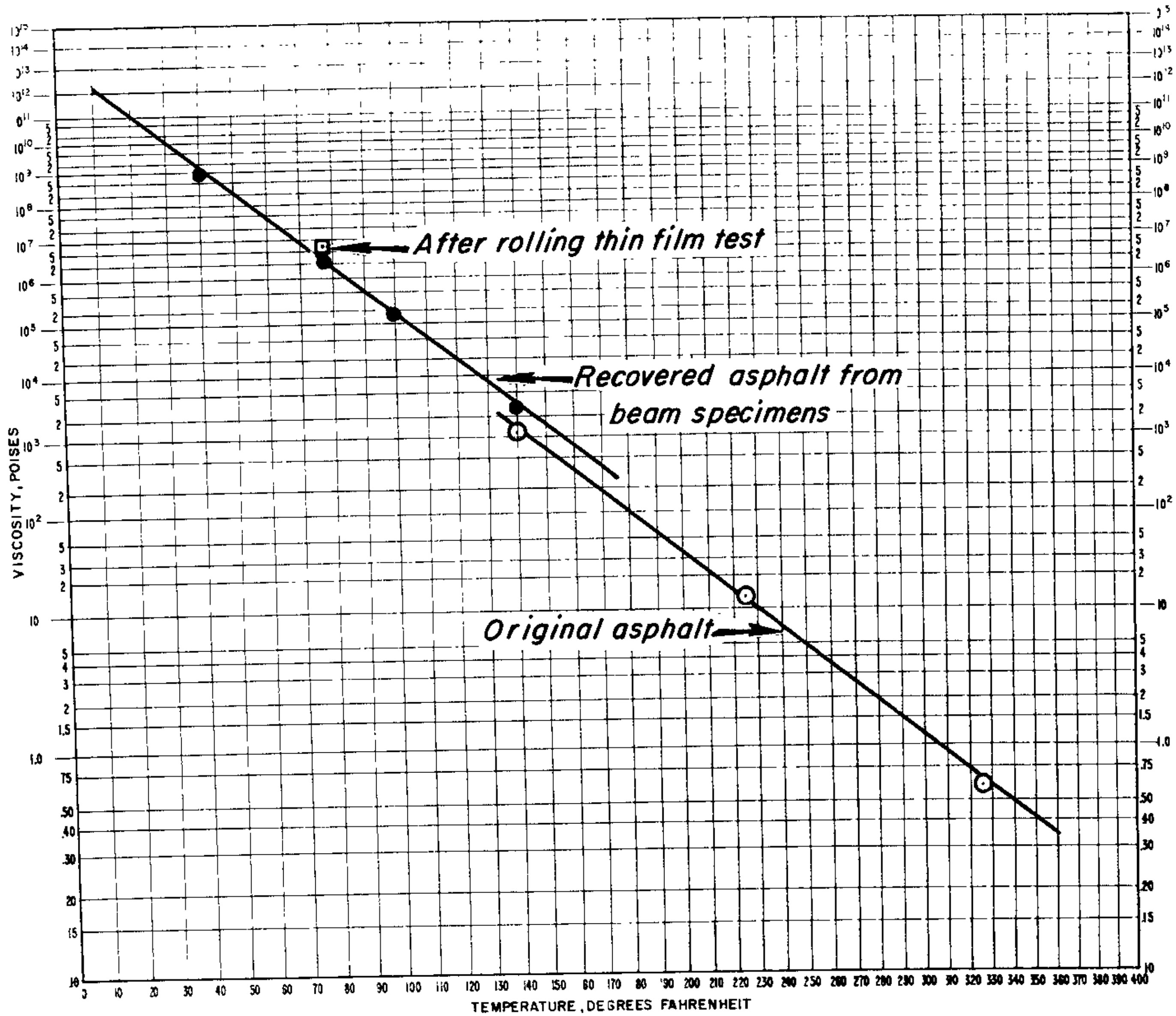


Fig. 15 - Viscosity-temperature relationships, G-series asphalt.

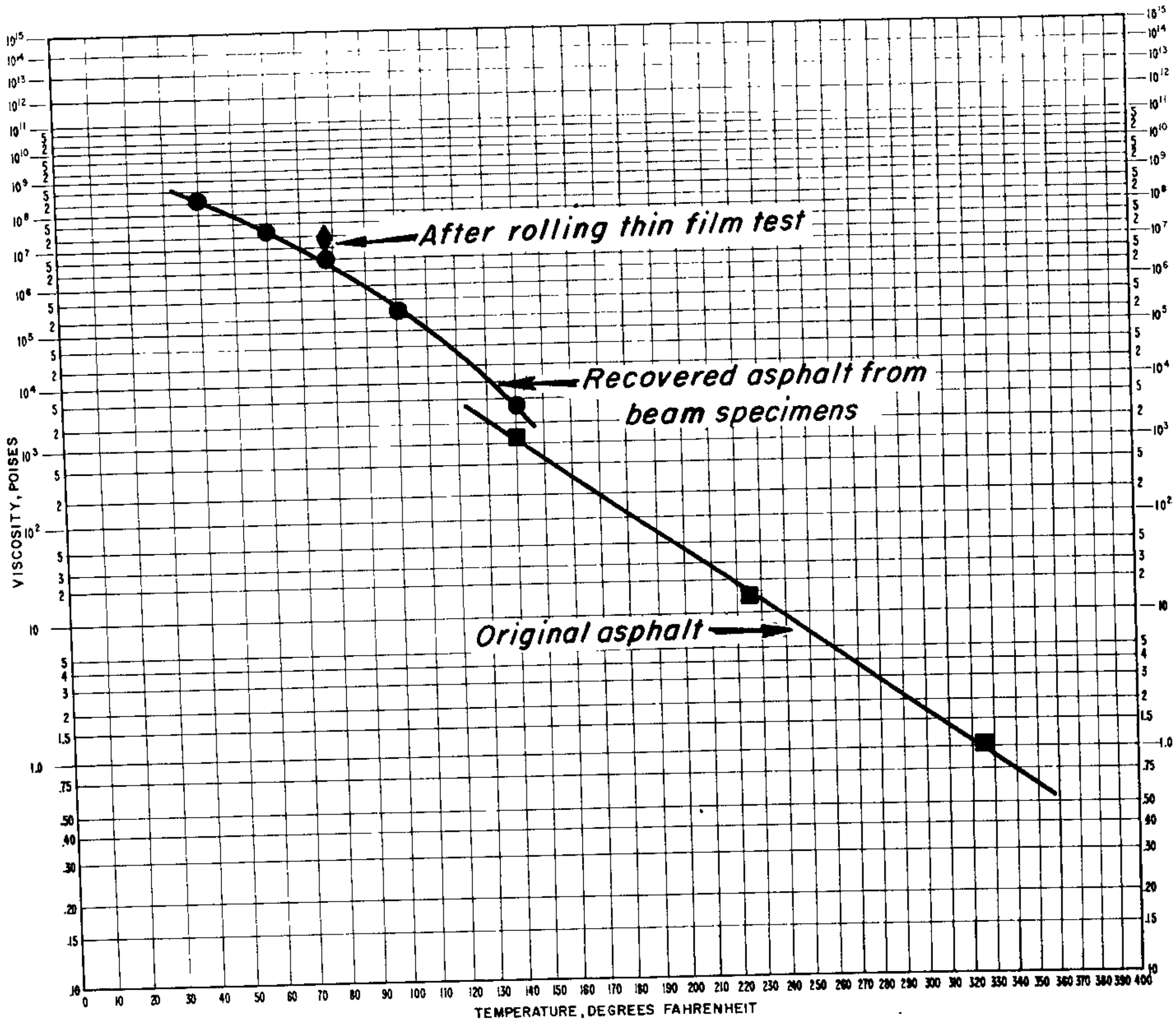


Fig. 16 - Viscosity-temperature relationships, E-series asphalt.

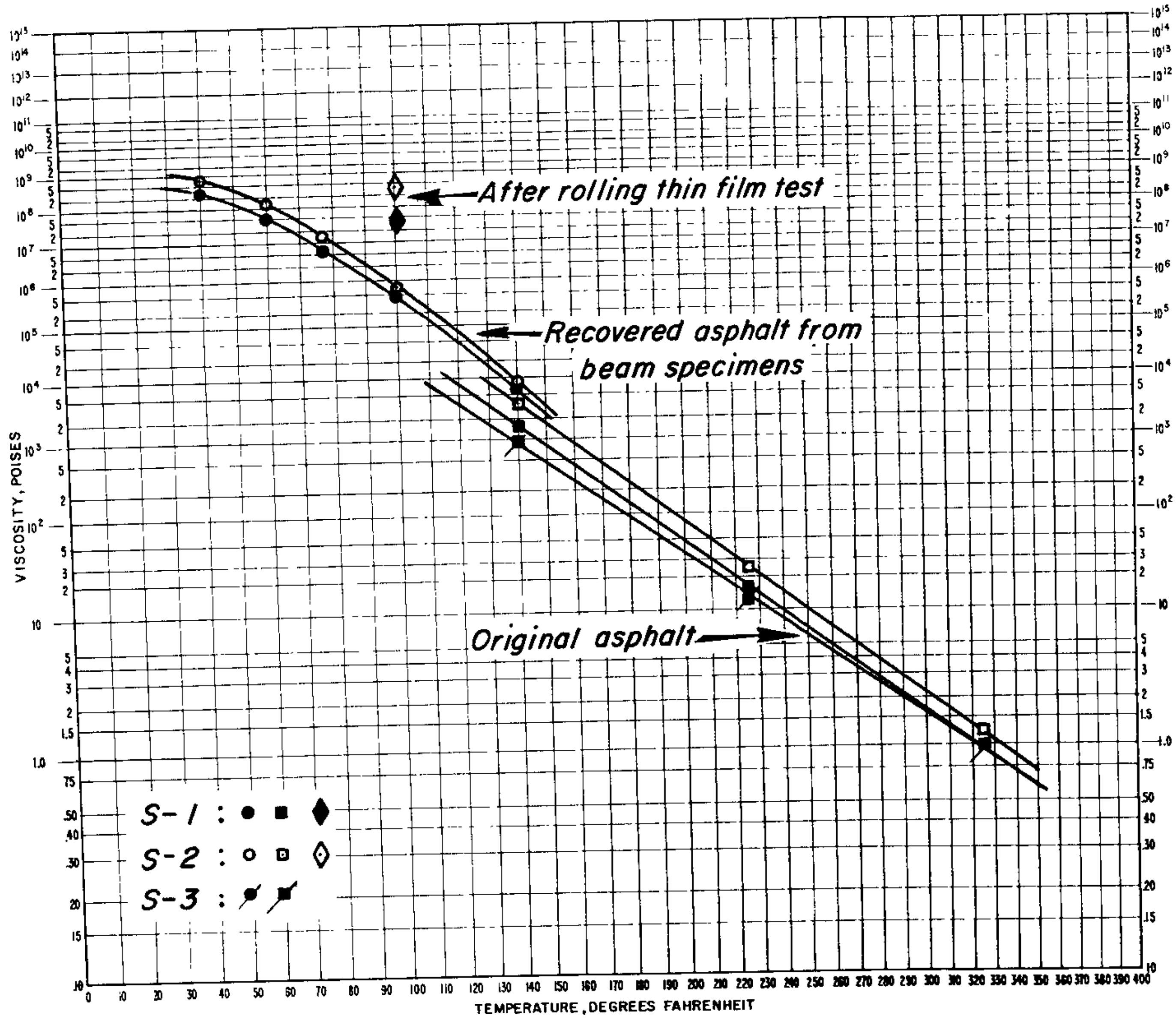


Fig. 17 - Viscosity-temperature relationships, S-series asphalts.

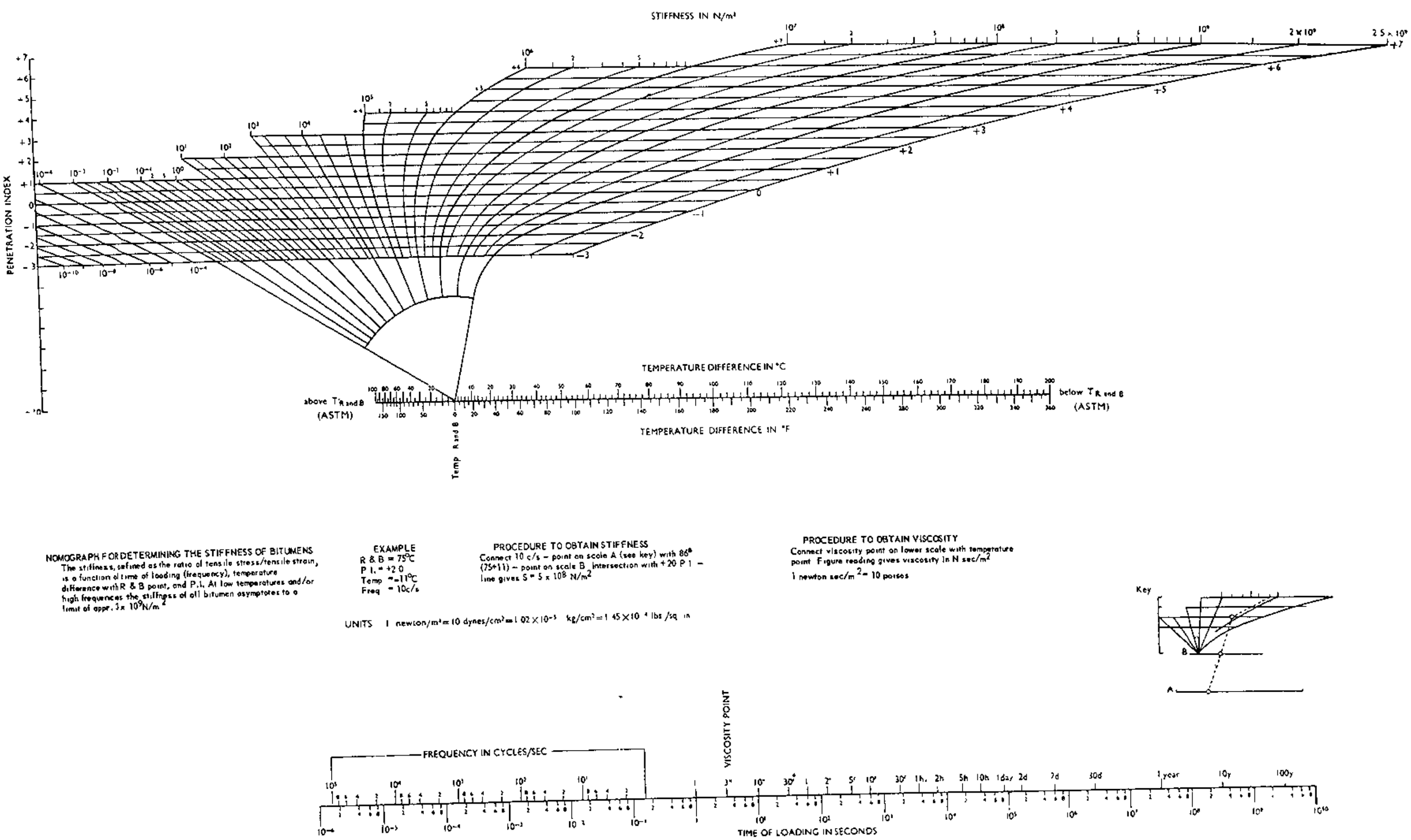


Fig. 18 - Nomograph for determining the stiffness of bitumens.

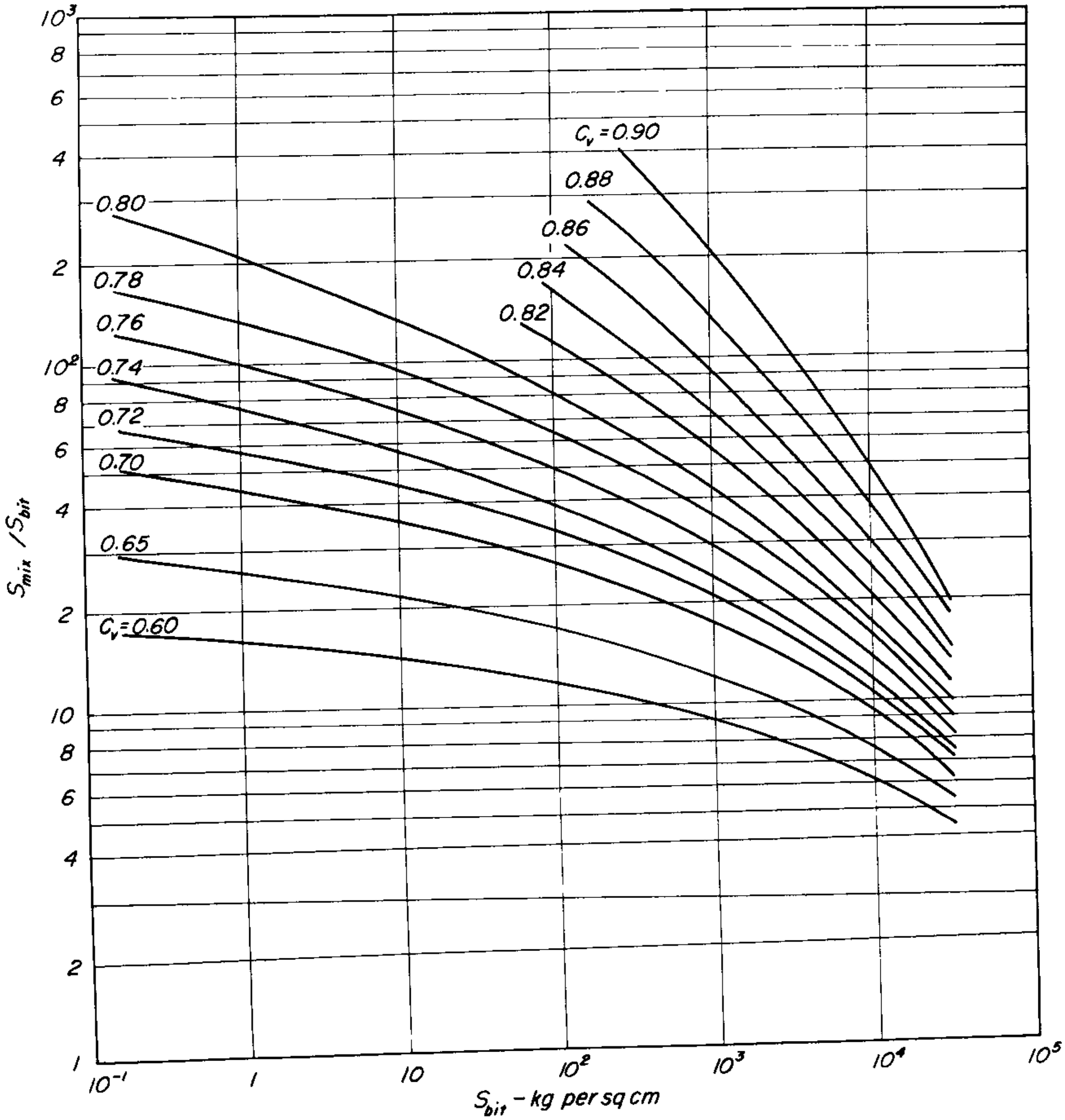


Fig. 19 - S_{mix}/S_{bit} as a function of S_{bit} and C_v. (After Heukelom and Klomp.)

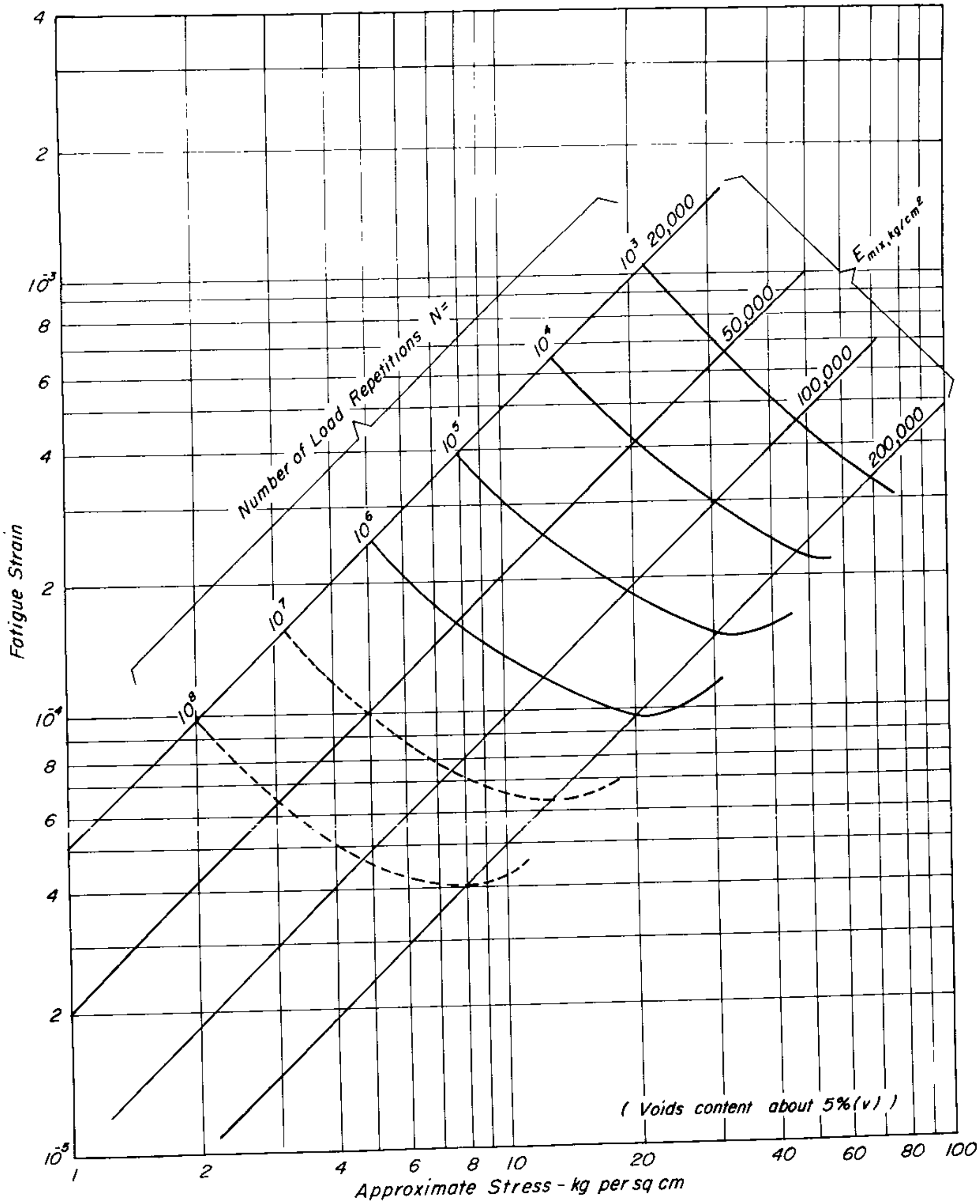


Fig. 20 - Bending fatigue strain upon repeated loading of bituminous base materials. (After Heukelom and Klomp.)

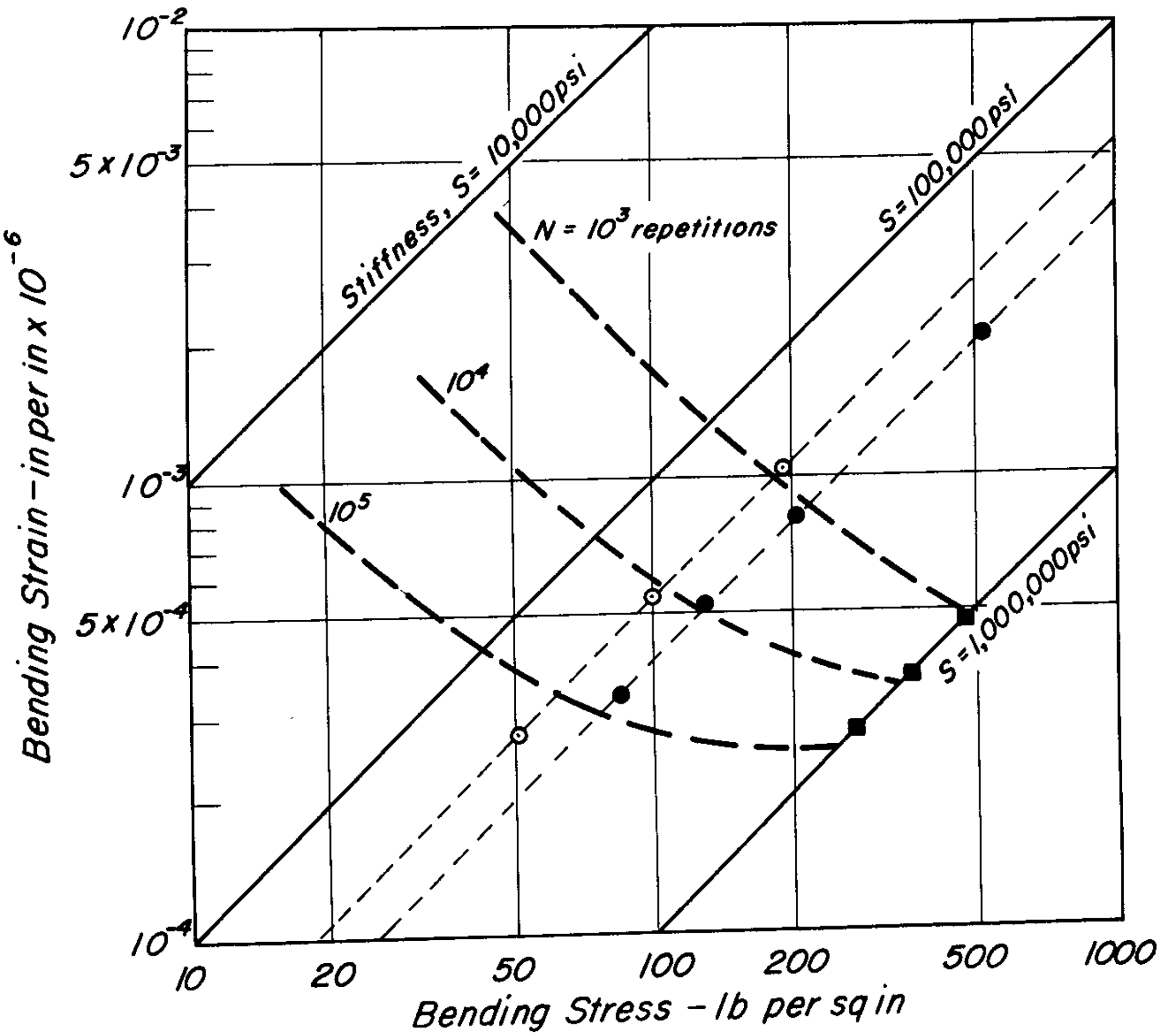


Fig. 21 - Relationship between stress, strain, and repetitions to failure in constant stress tests — Watsonville granite and 85-100 pen asphalt.